DESIGN OF ARCH RIB BRIDGE FOR NORTH SHERIDAN ROAD, WAUKEGAN, ILL.

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1911

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A THESIS PRESENTED BY

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and

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to the

PRESIDENT and FACULTY

OF THE

ARMOUR INSTITUTE OF TECHNOLOGY

for the degree of

BACHELOR OF SCIENCE IN CIVIL ENGINEERING

HAVING COMPLETED THE PRESCRIBED COURSE OF STUDY

IN

CIVIL ENGINEERING

Chicago, Ill., April 1, 1911.

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DESIGN OF PROPOSED ARCH RIB CONCRETE BRIDGE FOR NORTH SHERIDAN ROAD, WAUKEGAN, ILL.

The city of Waukegan, Ill., is cut by a large ravine varying in width from 200 feet to 350 feet, dividing the town in two. The main business street of the town, and North Sheridan Road cross the ravine, and both are carried by steel structures across it. The one on Sheridan Road is composed of three spans of 61 feet each resting on two towers. The outline of the structure is represented on the profile. It was erected about 25 years ago.

Owing to the profile of the street, the sloping sides of the banks made it difficult to determine the number and lengths of spans that would be practible. Anumber of tentative designs were considered, namely; (1) a central span of 130 feet with 50 foot approach spans, (2) two equal spans of 100 feet each, and (3) a single span of 150 feet with retaining wall approaches. The first was not considered because of the excess amount of form work necessary for the three arches, and the second was not considered desirable from an aesthetic view point. The single span was chosen because it would be the easiest to construct, more economical in form work and would have a more pleasing appearance when completed.

The feature of the design is the arch ribs. In a highway bridge, where the loads are not excessive, an arch ring of full width is not necessary. One of the advantages

in using ribs is that there is a great economy of concrete and the general appearance of a rib bridge is better than the ring bridge. One of the disadvantages of this kind of a bridge is that the form work is greater than on a ring bridge, but this is more than overbalanced by the economy of concrete. The width of roadway was taken as 36 feet from curb to curb with 8 foot sidewalks on each side of the roadway. The ribs are three in number, spaced 18 feet center to center, and the sidewalks are "cantilevered" out, making the entire width of the bridge 52 feet. This makes the load on each of the outer ribs about equal to that of the center, so that the three ribs may be made alike.

Method of Design,

In the analysis of the arch ribs, the elastic theory was used, following the method developed in Turneaure and Maurer's "Principles of Reinforced Concrete Construction." The maximum stresses allowable were as follows: concrete, local members, 600#/sq.in., main arch ribs, 600#/sq.in. including temperature; steel in the beams and girders, 15,000# main ribs 12, 000#/sq.in. The moduli of elasticity were as follows: steel, 30,000,000#; concrete, main members, 2,500,000# local members, 2,000,000#. The formulas used in the design of the slabs beams and girders were taken from the above book. The arch was analyzed for three for three different cases of loading; namely (1) liveload over the entire arch, (2) liveload on the middle third, and (3) live load on onehalf of the span plus the dead load in each case.

Loadings

The live loads were taken from curves given in Wadell's "DePontibus," a load of 1500# per linear foot being taken, extending over a width of 11 feet, making the live load per square foot of pavement equal to 136#. This live load was used over the entire floor, rather than to change it for the remaining 7 feet. The dead loads used are as follows: concrete with reinforcement, 150#/sq.ft., 4 inches of wood block pavement with a 2 inch sand cushion, 38#/sq.ft. A uniform load of 100#/sq.ft. was used on the sidewalk.

Method of Construction

ɪhe foundations are to be of 1 - 2 1/2 - 5 mixture, and all of the remainder of a 1 - 2 - 4 mixture. The ribs are to be erected in sections symmetrical about the center line of the bridge. The first two divisions to be put in are those at the springing line, then the one at the crown, and lastly the two on either side of the crown section. The end sections are to be built in pockets, left in the abutments, and are fastened to it by means of tie straps. The object of building it up in this manner is to keep the forms from bulging at the center, when loaded at the springing line and no load on the crown.

The general dimensions of the bridge are as follows: length of span, 150'- 0"; rise 22'- 0". The clear span is 148' - 0" and the overall length is 230'- 0". The span

was divided into 15 panels of 10'- 0" each. The abutment is constructed with an open core in the center, and is made the same length as the panel for the sake of uniformity. The approach fill is supported between two retaining walls resting directly on the masonry foundation, and is connected to it by means of rods extending from the masonry.

Design of Floor Slab,

The floor slab was made 10 feet long by 18 feet wide. The reinforcing bars were run only in the longitudinal direction, the entire load being assumed to be carried to the transverse floor beams which were placed 10 feet center to center.

From "De Pontibus", an equivalent live load of 1500# per linear foot of track was found to fulfill the required conditions. This was assumed to be spread over a distance of 11 feet, the distance between track centers thus giving a liveload of 136# per sq.ft. over the car tracks, and the same value was used over the entire width of slab.

A floor slab 8 inches thick was first assumed. On top of this there is to be placed a two inch sand cushion, the foundation for the wood block pavement. The load on one foot of slab 10 feet long was 2,863# which gave a bending moment of 43,000 inch pounds (beam being designed as a simple beam). Taking the allowable working stress of concrete

as 600#/sq.in., a depth of 6" was required to keep the concrete within the working stresses, thus making the total depth 8"; 2" of concrete being assumed from the center of the steel to the bottom of the slab. From the formula

$$A = \frac{M_s}{7/8d \cdot f_s}$$

where A = number of square inches of steel, M_s = bending moment taken by the steel, d = depth of the beam, and f_s = 12,000#. The number of square inches of steel required per foot of slab was determined and 11/16" rods spaced 6" apart were found to be sufficient to take care of the tensile stresses.

Design of Floor Beam,

The transverse floor beam was also designed as a simple beam. The uniform load per linear foot of beam was taken as the load per linear foot of slab plus the load due to the weight of concrete in the assumed web of the beam. The area of the beam was made sufficient of the vertical shear. Two hundred and seventy-three square inches were required allowing 100#/sq.in. for the concrete in shear. A web 12"x 24" (to center of steel) was found to be the most desirable considering the spacing of the steel. This brought the neutral axis in the web of the beam, thus making it necessary to design it as a T-beam. From Plate IX. page 283, of Turneaure and Maurer's Reinforced Concrete Construction, f_s being taken as 15,000, M/bd^2 was found

to be 98, d being known, the value of b was solved for and found to be satisfactory. From the curves for j, the value of jd was found. Everything was now known that was necessary to determine the required steel area. Eight 7/8" rods were found to be necessary to take care of the tensile stresses. The rods were spaced in two rows, four in a row, three inches apart. It was found necessary to run four rods straight through to develope the maximum possible bond stress. The necessary lengths of the rods to resist the bending moment was found from the formula

$$X_n = \left(l/\sqrt{A}\right)\left(a_1 + a_2 + a_3 \ . \ . \ .\right)^{\frac{1}{2}}$$

where

X_n = length of the rod

l = length of span

A = area of the steel

a_1, a_2, and a_3 are the areas of the respective rods

The following lengths were found to be required; 1st, 6.36'; 2d, 9.00'; 3d, 11.00'; 4th, 12.70'.

Inorder to make a good design, the first two rods were turned up so that their ends were over the support. The next two were turned up 2 feet further. The stress per square inch in the rods for the worst possible case was found to be 11,700#, allowing 30#/sq.in. to be taken by the concrete.

The remainder of the shear is carried by 3/8" stirrups used in the form of a double loop, and spaced 10" apart up

to a point 2 feet from the center, and from there on the spacing is to be 18" apart. Stirrups are also placed 18" apart between the bent up rods.

Sidewalk Slab and Floor Beam.

The depth of the sidewalk floor slab was made the same thickness as the roadway floor slab (8") for sake of uniformity. This only required a steel area of .571 sq.in., 5/8" rods spaced 6" apart were therefore sufficient to take up the tensile stress.

The side walk floor beam is a cantilever. The maximum bending moment will therefore occur at the point of support. For simplicity, the beam was made 12" wide and 26" deep at the support, the same as the roadway floor beam. The steel reinforcing in this case was put in the top of the beam. . Four 7/8" rods and four 5/8" rods were used, a steel area of 3.6 sq.in. being required.

DESIGN OF MAIN ARCH RIB.

The method of analysis used in the design of the arch was based on the elastis theory as given in Turneaure and Maurer's "Principles of Reinforced Concrete Construction." A rib 4 feet in width varying from four feet in depth at the crown to five feet at the springing line was assumed. The method was analytical throughout with the exception of the determination of the lengths of divisions and the thrusts which were ontained graphically.

The arch was first divided into ten equal divisions. A steel area of 12 sq.in. was assumed, which is the equivalent of about .5% reinforcement. The combined moment of inertia of the concrete and steel were then figured at each of the ten sections, a section being taken at the middle of each division. From the values thus obtained, the average moment of inertia was dertermined.

It was necessary the rib into divisions of such length that "ds/I" is constant, where "ds" is the length of divisions measured along the neutral axis and "I" is the combined moment of inertia of the concrete and steel. The value of ds/I is equal to $(s \cdot i_a/n)$, where s= half the length of the arch measured along the axis, i_a = the average combined moments of inertia and n = the number of divisions in one half of the arch. A figure was drawn whose base was equal to s and divided into ten equal parts. At the center of each division, the corresponding reciprocal of the combined moments of inertia were laid off as ordinates to some convenient scale. The ends of the ordinates were connected by a smooth curve. The area then enclosed was then divided into ten equal parts, the resulting ordinates of each division of area was the reciprocal of the true moment of inertia of each section, and the base the correct length of each division.

The center line of the rib was divided into the lengths thus obtained and the corresponding abscissas and ordinates from the center of the section scaled off, using

the crown as origin. The bending moment due to external loads at each section were figured. As the bridge was divided into panels of ten feet each in length, there were consequently seven external loads on each half of the arch to be considered. In the first case where the bridge was assumed to be entirely covered with live load, each external load on the rib was a constant plus the weight of concrete in the rib between the columns, the concrete being assumed as concentrated at these points. As the loads were symmetrical about the center, only one half of the arch had to be considered in calculating the stresses. The thrusts at the crown were calculated from the formula

$$H_o = \frac{n\Sigma my - \Sigma m \quad y}{2\left((\Sigma y)^2 - n\Sigma y^2\right)}$$

where

n = number of divisions in one half of the arch,

m = bending moment at any point in the arch due to external loads

x & y = ordinates at any point on the rib axis referred to the crown as origin, and all considered as positive in sign.

As the summation of the bending moments on the right equalled the summation of those on the left of the rib, the shear at the crown was zero. The bending moment at the crown was calculated from the formula

$$M_o = -\frac{\Sigma m + 2H_o\Sigma y}{2n}$$

After having determined these values, the total bend-

ing moment at each section was calculated from the formula

$$M = m + M_o + H_o y \pm V_o x$$

where

M = total bending moment at any section,

m = bending moment at any point due to the external loads,

M_o = bending moment at the crown, assumed as positive when causing compression in the upper fibres,

$H_o y$ = bending moment at the crown due to the thrust,

$V_o x$ = bending moment at the section at the section due to the shear at the crown.

The bending moment due to the shear at the crown is considered positive for the left half of the arch, and negative for the right half.

By means of a graphical diagram, the thrusts at each section were determined. The loads were laid off to scale on a vertical load line, the thrust at the crown was then laid off to the same scale in a horizontal direction at a distance above or below the junction of the loads adjacent to the crown, equal to the shear at the crown. If the shear was negative, it was laid off downward from the junction, if positive upward. By joining the pole (the extremity of the horizontal line) with the points of division on the load line, the thrusts at each section could be readily scaled. This was assumed to be the true thrust, the shear at the section being negligible.

The eccentricity at any section could now be readily calculated; being equal to the quotient of the bending moment divided by the thrust.

In the second analysis, the middle third of the arch was considered as covered with live load. The same operations were repeated as in the first case. The shear at the crown was again zero as the bending moments on each side of the crown were equal.

In the third analysis, the left half of the arch was assumed to be covered with live load. This condition produced shear at the crown, as the bending moments on the left side of the crown were greater than those on the right. The same operations were repeated in a similar manner to the first two cases.

These three conditions of loading were all that were considered necessary, as the greatest possible fibre stresses would be produced under this variety of loading.

Temperature Stresses.

The arch was designed to be constructed at an average degree of temperature. Stresses were then figured for a variation of thirty degrees Fahrenheit in temperature above and below that at which the arch was constructed. The thrust at the crown due to this variation of temperature was calculated from the following formula:

$$H_o = \frac{EI}{ds} \frac{c\,t\,l\,n}{2(n\Sigma y^2 - (\Sigma y)^2)}$$

where

E = moduli of elasticity of concrete, taken here as 2,500,000

I/ds = constant = .22908

c = coefficient of expansion = .000006

t = rise or fall of temperature in degrees

l = length of spann = 150'.

n = number of divisions in one half of the arch.

The summations of the y and y^2 refer to onehalf of the rib only. The bending moment at the crown is equal to:

$$M_o = -\frac{H_o \Sigma y}{n}$$

The bending moment at any section of the arch was then calculated from

$$M = M_o + H_o y$$

The thrust and shear at all sections were then determined by resolving the thrust at the crown parallel and normal to the rib axis at that point, this being done graphically.

Fiber stresses.

The fiber stresses were calculated from the following formula

$$f_c = M_c/I + N/A$$

where

M = bending moment at the section,

c = distance to the outer most fiber,

I = combined moment of inertia,

N = thrust on the section,

$A = (A_c + 15A_s)$ transformed area, A_c = area of concrete, A_s = steel area.

The maximum fiber stresses at each point and under what condition of loading are given in one of the following tables. The fiber stress due to temperature changes were combined with those due to the loading.

The maximum resultant stresses were all found to be too high. The steel area was consequently increased to 48 square inches and the fiber stresses recomputed. It was unnecessary to re-analyze the arch rib, as the moment of inertia of each section was increased in the same ratio. This time, it was only necessary to calculate the fiber stresses for the loading which produced the maximum stress, this being done previously for an increase of steel area would not change the condition of loading which produce the maximum stress in a member.

The maximum fiber stresses all fell within 600#/sq,in. except at point 10 and the springing line. This was reduced by increasing slightly the area of concrete by raising the extrados six inches at the springing line, thus giving the rib a depth of 5' - 6" at that point. This produced a maximum fiber stress at point 10 of 607#/sq.in. which is

allowable. The fiber stress at the springing line is also within the limits as it occurs in the abutment. The arch was rounded off at the abutment so as to it an aesthetic appearance,mwhich at the same slightly increases its strength.

Design of the Abutment.

The maximum thrust at the abutment occurs when the bridge is fully loaded and amounts to 1,131,000 pounds per rib. This is applied at an angle whose

$$\tan^{-1} \frac{138.84 - 22}{75} \qquad \phi = 57^{\circ}\ 20'.$$

Trials were made graphically until the resultant of the thrust and the weight of the abutment plus the earth fill above came well within the middle third. The abutment was made 42' wide and built in three different heights as shown in one of the following blueprints. Concrete was figured at 150# per cubic foot, and earth as 120# per cubic foot.

Design of Retaining Wall.

The purpose of the retaining wall is to support the earth fill between the abutments and the end of the bridge. Where the wall is the highest, it will rest directly on the masonry and fastened to it by enough rods to overcome the effect of sliding. The top of the walls will be tied together by means of rods extending through the floor slab, and the wall itself will be designed as a simple beam to

take up the horizontal pressure of the earth.

One linear foot of wall will be considered only, and a thickness of 30" will be assumed for trial thickness. As the top of the wall is tied, there will be no advantage in making the wall thicker at the bottom than at the top.

The horizontal pressure of a column of earth acting at the center of gravity is given by the formula

$$P = 1/6\ W\ h^2$$

where W is the weight of earth, and h is the height of the column. For a column of earth 23 feet in height, the horizontal pressure is 10,580# acting horizontally at a distance of 7 2/3 feet up from the base. Taking moments about the base to find the stress which will be developed in the tie rods of the floor slab, it comes out to be 4,060#, and .25 sq.in. of steel per foot will be required. 13/16" round rods spaced every two feet will be used. To find the amount of steel area required to take up the shear moments were taken about the steel in the floor slab. The stress to be overcome is 7,080# requiring .708 sq.in. and in this case, 1" rods spaced a foot apart will be used.

Considering the wall as a simple beam with a load of 10,580#, a depth of 29" was required. The wall was made 30" thick, 1 inch being allowed for the steel. The area of steel required was 1.2sq.in. per foot, and 3/4" round rods will be used spaced 6" center to center.

Design of the Falsework.

The water way averages about 8 feet in width and no where a foot deep in the low water season, and the banks are perfectly dry, so there would be no trouble in driving piles for the falsework to rest on. The bents were spaced on 15 foot centers in the middle of the arch, narrowing down towards the abutments to 10, 8, and 6 feet respectively. Each bent is composed of 6 piles in three pairs, the pairs being 18 feet apart, each pair coming under on of the ribs. The bents are capped with 12" x 12" timbers, and on top of these are 6 - 12"x12" running lengthwise of the bridge. On top of these is another row of bents 10 feet in height each being capped with a 12"x12" at right angles to the bridge. The wedges are placed on top of the caps, and another 12"x12" on top of the wedges.

The ring was divided into 5 foot divisions, and props were erected to support the lagging at about 5 foot intervals. Taking the worst condition, that of the middle inclined strut, the perpendicular load coming on it is 6,500#, half of the weight of one of the 5 foot divisions. Gordons formula was used in determining the cross-section.

$$P = \frac{f\,S}{1 - \frac{l^2}{125\,h^2}}$$

Where

P = direct load

f = allowable stress (1300# being taken in this case for Geogia pine)

l = length of the member in inches

S = area of cross-section

h = least diameter

The area reqired was 68 sq.in. and a 6"x12" having an area of 72 sq.in. was used.

For the verticals, the load used was 19,500#, 137 sq.in. were required and a 12"x12" was used. The 12"x12" are to come in 24 foot lengths, and to be cut as needed.

The cross- bracing was made up of entirely 3"x12" for uniformity. The stresses in the cross bracing are indeterminate, and the number of braces will be governed by past experience.

DIVISIONS OF ARCH RIB.

Properties of Preliminary Equal Divisions.						Properties of Final Divisions.				
No. of Division	Depth	I_c	$15 I_s$	$I = I_c + 15 I_s$	$i = \frac{1}{I}$	ds	i	I	I_c	Depth
1	4.05	22.14	3.94	26.08	.0383	5.91	.0388	25.77	21.83	4.04
2	4.15	23.82	4.16	27.98	.0357	6.25	.0367	27.25	23.09	4.11
3	4.25	25.59	4.39	29.98	.0333	6.63	.03455	28.94	24.55	4.18
4	4.35	27.44	4.63	32.07	.0312	7.00	.03275	30.53	25.90	4.26
5	4.45	29.37	4.88	34.25	.0292	7.45	.03075	32.52	27.64	4.36
6	4.55	31.40	5.12	36.52	.0274	7.96	.0288	34.72	29.60	4.46
7	4.65	33.51	5.38	38.89	.0257	8.50	.02695	37.11	31.73	4.56
8	4.75	35.72	5.64	41.36	.0242	9.10	.0252	39.68	34.04	4.67
9	4.85	38.03	5.91	43.94	.0227	9.85	.0233	42.92	37.01	4.80
10	4.95	40.43	6.19	46.62	.0214	10.59	.02165	46.19	40.00	4.93
Total					.2891	79.24				
Springing	5.00	41.67	6.34	48.00						

$i_a = .2891/10 = .02891$ $\qquad$ $ds.\,i = \frac{79.239 \times .02891}{10} = .22908$

Length of the Final
areas = ds.i.

Dotted Lines denote the
Preliminary Divisions.

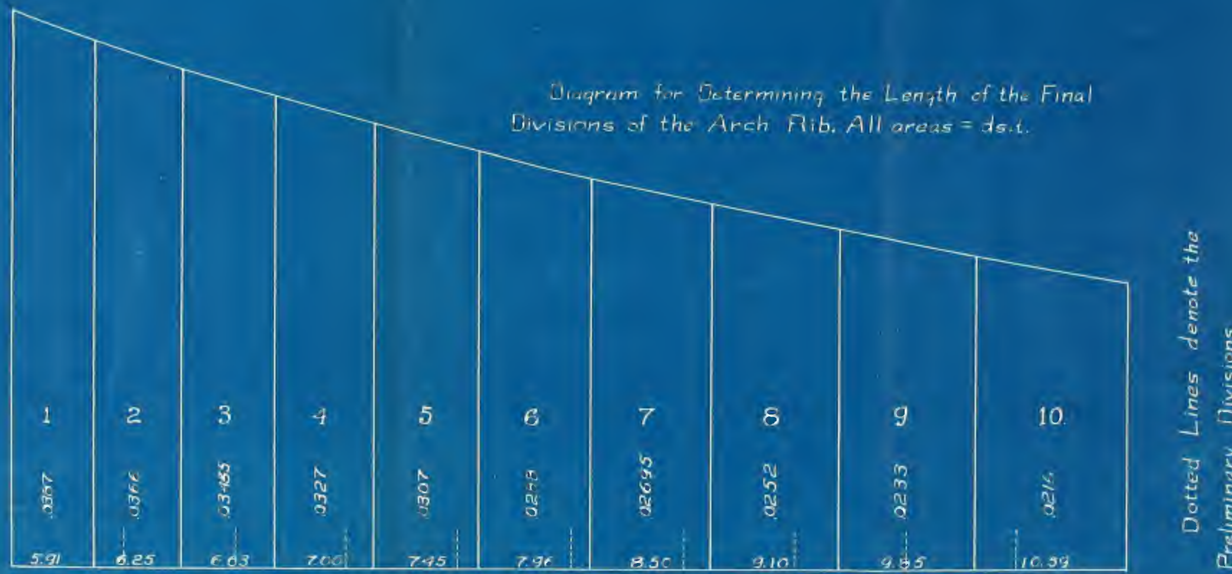


Diagram for Determining the Length of the Final Divisions of the Arch Rib. All areas = ds.i.
1
2
3
4
5
6
7
8
9
10.
.0387
.0366
.03485
.0327
.0307
.0288
.02695
.0252
.0233
.0216
5.91
6.25
6.63
7.00
7.45
7.96
8.50
9.10
10.39
Dotted Lines denote the Preliminary Divisions.

CASE 1

Arch covered with live load.

Point	x.	y	x^2	y^2	m_L or m_R	$m_L + m_R$	$(m_L+m_R)y$	Thrusts Right or Left	H_0y	Bending Moment	Eccentric Distances
1	2.90	.2	8.41	.04	- 0.0	- 0.0	0.0	994,800	+198,600	+11,260	+.0113
2	9.02	.4	81.36	.16	-297,500	-595,000	238,000	996,000	+397,920	-87,280	-.0876
3	15.42	1.0	238.63	1.00	-803,600	-1,607,200	1,607,000	1,005,000	+994,800	+3,500	+.0035
4	22.2	2.0	492.84	4.00	-1,813,000	-3,626,000	7,252,000	1,005,000	+1,989,600	-11,100	-.0111
5	29.4	3.6	864.36	12.96	-3,244,500	-6,489,000	23,350,000	1,019,600	+3,580,000	+147,800	+.1450
6	36.84	5.2	1357.27	27.04	-5,050,300	-10,100,600	52,600,000	1,039,600	+5,170,000	-68,000	-.0655
7	44.7	7.7	1998.09	59.29	-7,421,000	-14,842,000	114,300,000	1,039,600	+7,555,000	-53,700	-.0516
8	52.96	10.7	2804.86	114.49	10,534,000	-21,068,000	225,800,000	1,064,000	+10,630,000	-91,700	-.0862
9	61.6	14.4	3794.56	202.36	-14,334,000	-28,668,000	413,000,000	1,095,000	+14,310,000	-211,700	-.1930
10	70.6	19.4	4984.36	326.36	-18,909,000	-37,818,000	735,000,000	1,131,000	+19,300,000	+203,300	+.1798
Σ	345.64	64.6	16624.74	802.7	-62,406,900	-124,813,800	1,573,147,000				
Spring	75.0	22.0	5625.0	484.0	-21,279,000	-42,558,000		1,131,000	+21,870,000	+403,300	+3560

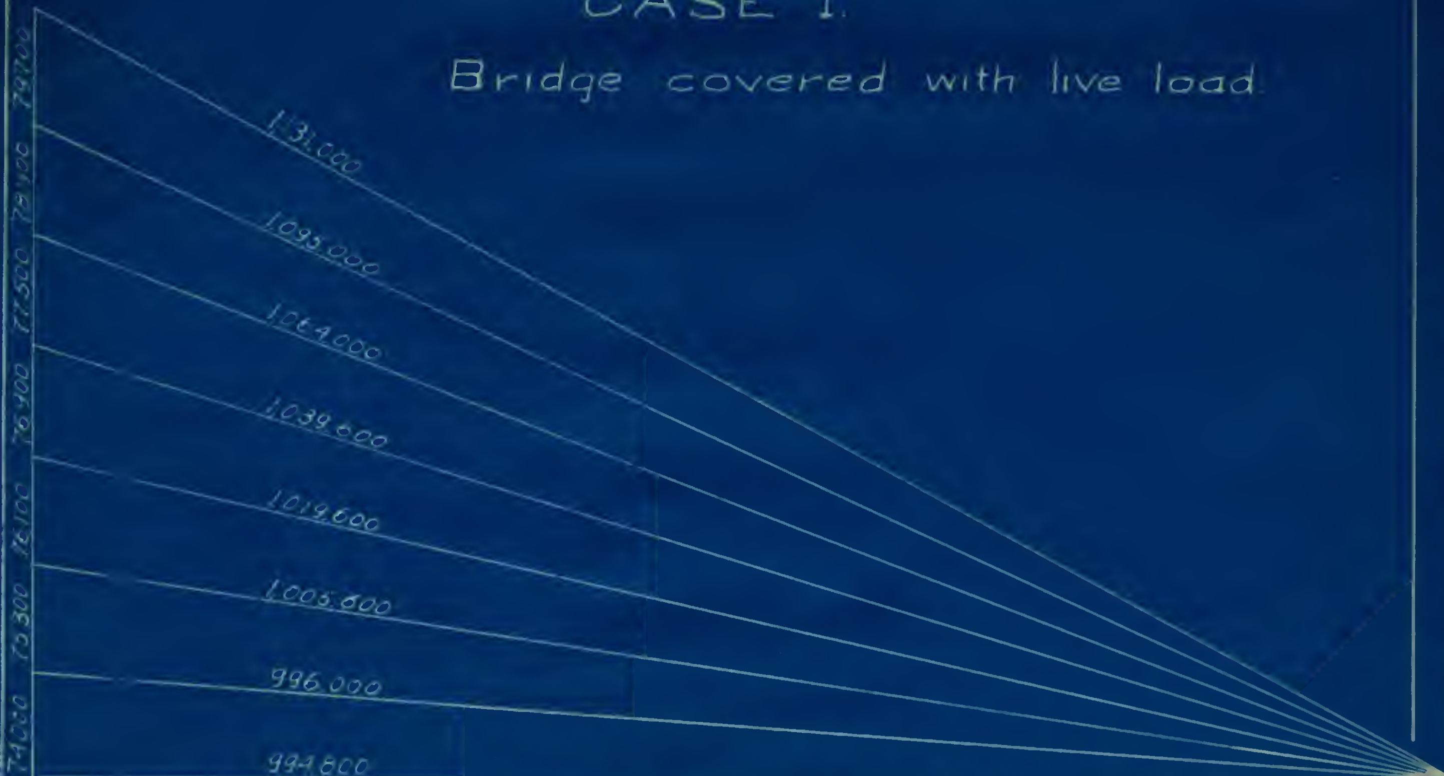
$$H_0 = \frac{10(1,573,147,000) - (-124,813,800)64.6}{2[(64.6)^2 - 10 \times 802.7]} = +994,800$$

$$V_0 = 0.0$$

$$M_0 = -\frac{(-124,813,800) + 2(994,800 \times 64.6)}{20} = -187,700$$

CASE 1.

Bridge covered with live load



$H_o = 994800$

CASE 2

Middle Third of Arch Loaded

Point	X	Y	X^2	Y^2	m_L or m_R	$m_L + m_R$	$(m_L+m_R)Y$	Thrusts Right or Left	H_oY	Bending Moment	Eccentric Distances
1	2.90	.2	8.41	.04	0.0	0.0	0.0	892,000	+178,200	+240,440	+.269
2	9.02	.4	81.36	.16	−297,500	−595,000	238,000	895,000	+357,000	+121,740	+.136
3	15.42	1.0	238.63	1.00	−803,600	−1,607,200	1,607,200	905,000	+892,000	+150,640	+.166
4	22.2	2.0	492.84	4.00	−1,813,000	−3,626,000	7,252,000	905,000	+1,784,000	+33,240	+.037
5	29.4	3.6	864.36	12.96	−3,244,000	−6,488,000	23,350,000	919,000	+3,210,000	+28,240	+.031
6	36.84	5.2	1357.27	27.04	−5,005,500	−10,011,000	52,057,000	934,000	+4,640,000	−303,260	−.325
7	44.70	7.7	1998.09	59.29	−7,185,000	−14,370,000	110,800,000	934,000	+6,870,000	−252,760	−.271
8	52.96	10.7	2804.86	114.49	−9,896,000	−19,792,000	212,000,000	952,000	+9,540,000	−293,760	−.308
9	61.6	14.4	3794.56	202.36	−13,114,000	−26,228,000	378,000,000	971,000	+12,830,000	−221,760	−.228
10	70.6	19.4	4984.36	326.36	−16,887,000	−33,774,000	655,000,000	995,000	+17,300,000	+475,240	+.478
Σ	345.64	64.6	16624.74	802.7	−58,245,600	−116,491,200	1,440,304,000				
Springing	75.0	22.0	5625.0	484.0	−18,829,000	−37,658,000	828,000,000	995,000	+19,610,000	+843,240	+.848

$$H_o = \frac{10(-1{,}440{,}304{,}000) - (-116{,}491{,}200)\,64.6}{2[(64.6)^2 - (10 \times 802.7)]} = 892{,}000$$

$$V_o = 0.0$$

$$M_o = -\frac{(-116{,}491{,}200 + 2 \times 892{,}000 \times 64.6}{20} = +62{,}240$$

CASE 2

Middle Third Loaded

$H_0 = 892,000$

CASE 3

Left Half of Arch Loaded

Point	X	Y	X^2	Y^2	m_R	m_L	m_R+m_L	$(m_R+m_L)Y$	(m_R-m_L)	$(m_R-m_L)X$
1	2.90	.2	8.41	.04	0.0	0.0	0.0	0.0	0.0	0.0
2	9.02	.4	81.36	.16	−199,000	−297,500	−496,500	−199,000	+98,500	+889,000
3	15.42	1.0	238.63	1.00	−537,000	−803,600	−1,340,600	−1,340,600	+266,600	+4,110,000
4	22.2	2.0	492.184	4.00	−1,217,000	−1,813,000	−3,030,000	−6,060,000	+596,000	+13,218,000
5	29.4	3.6	864.36	12.96	−2,178,000	−3,244,500	−5,422,500	−19,500,000	+1,066,500	+31,323,000
6	36.84	5.2	1357.27	27.04	−3,398,000	−5,050,300	−8,448,300	−43,900,000	+1,652,300	+60,802,000
7	44.7	7.7	1998.09	59.29	−5,003,000	−7,421,000	−12,424,000	−95,700,000	+3,418,000	+152,610,000
8	52.96	10.7	2804.86	114.49	−7,111,000	−10,534,000	−17,645,000	−188,900,000	+3,423,000	+181,300,000
9	61.6	14.4	3794.56	202.36	−9,682,000	−14,433,400	−24,115,400	−347,800,000	+4,751,400	+292,600,000
10	70.6	19.4	4984.36	326.36	−12,809,000	−18,909,000	−31,718,000	−616,000,000	+6,100,000	+430,680,000
Σ	345.64	64.6	6624.74	802.7	−42,134,000	−62,506,300	−104,640,300	−1,324,399,600		+1,167,592,000
Springing	75.0	22.0	5625.0	484.0	−14,415,000	−21,279,000				

$$H_o = \frac{10(-1,324,399,600) - (-104,640,300)\,64.6}{2[(64.6)^2 - 10 \times 802.7]} = 841,200$$

$$V_o = \frac{+1,167,592,000}{2 \times 16,624.74} = 35,117$$

$$M_o = -\frac{(-104,640,300 + 2 \times 841,200 \times 64.6}{20} = -196,490$$

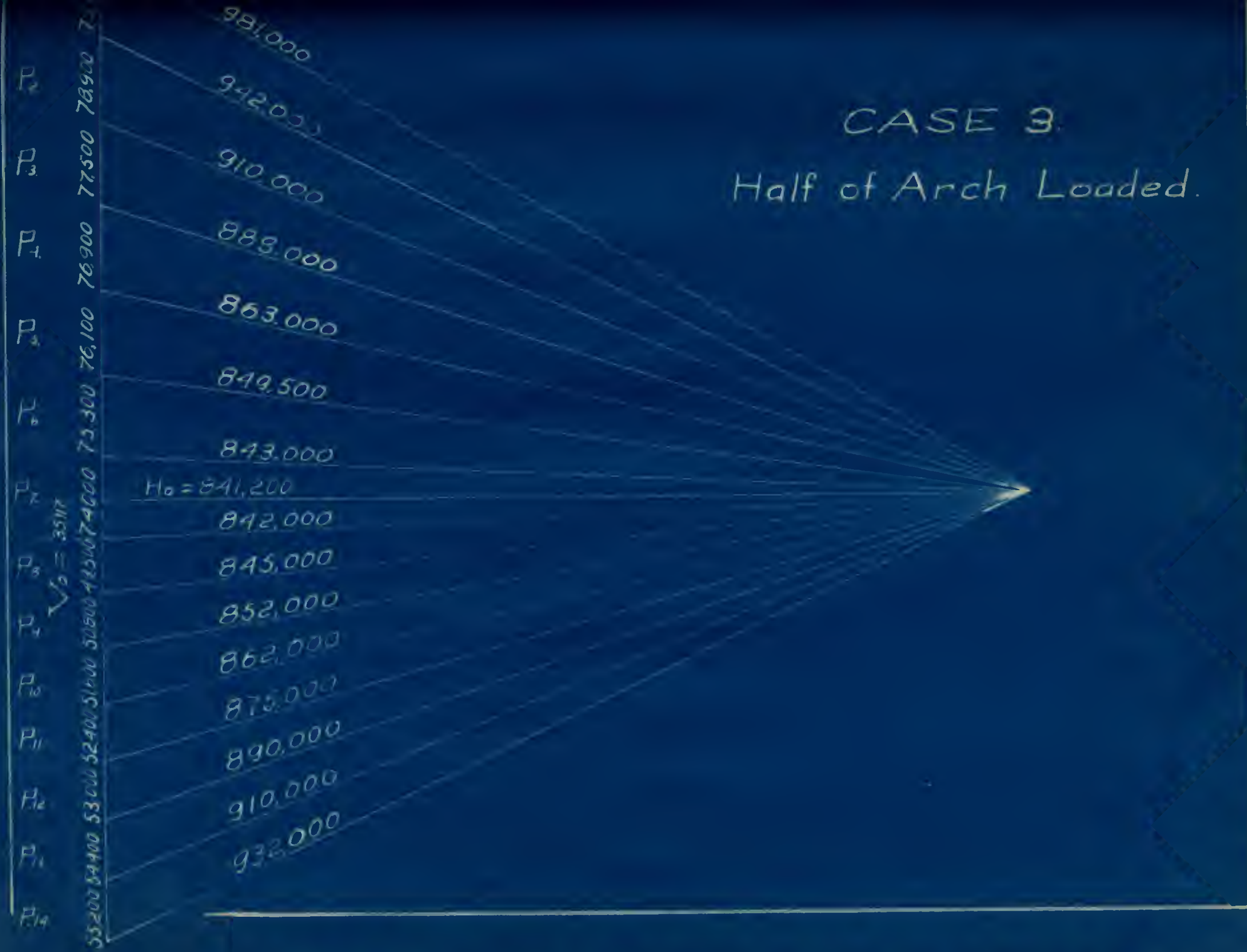
CASE 3 continued.

Left Half of Arch Loaded.

Point	Thrust Right	Thrust Left	$V_o x$.	$H_o y$.	Bending Moment R.	Bending Moment L.	Eccentric Distance R.	Eccentric Distance L.
1	842,000	842000	101,830	168,240	- 130,080	+ 73,580	-.155	+.087
2	845,000	843,900	316,640	336,480	-375,650	+159,130	-.445	+.189
3	852,000	849,500	541,260	841,200	-433,550	+382,370	-.508	+.451
4	852,000	849,500	779,600	1,682,400	-510,690	+452,510	-.600	+.532
5	862,000	863,000	1,032,400	3,028,300	-378,590	+619,800	-.438	+.718
6	875,000	883,000	1,293,800	4,374,300	-513,990	+421,310	-.587	+.477
7	875,000	883,000	1,569,600	6,473,700	-295,390	+425,810	-.338	+.483
8	890,000	910,000	1,859,800	9,000,200	-167,090	+129,510	-.188	+.142
9	910,000	942000	2,166,000	12,114,000	+65,910	-349,890	+.072	-.371
10	932000	981000	2,479,100	16,319,000	+834,410	-307,390	+.895	-.312
[illegible]	932000	981000	2,633800	18,507,000	+1,261,710	-334,690	+1.35	-.341

CASE 3

Half of Arch Loaded.



Conditions of Loading which produce the maximum fiber stresses in the concrete and the corresponding quantities necessary for their calculation; steel area being changed to 48 sq. in

Point.	Case.	M.	u.	I_c.	$15 I_s$	I.	N.	A_c.	$15 A_s$.	A.	M—Temperature	—N
1.	II	+240,440	2.02	21.83	15.76	37.59	892,000	16.16	5.00	21.16	-344,000	55,000
2.	III_R.	-375,650	2.055	23.09	16.64	39.73	845,000	16.44	"	21.44	-333,000	54,700
3.	III_R.	-433,550	2.09	24.55	17.56	42.11	852,000	16.72	"	21.72	-300,000	54,500
4.	III_R	-510,690	2.13	25.90	18.52	44.42	852,000	17.04	"	22.04	-245,000	54,300
5.	III_L.	+619,800	2.18	27.64	19.52	47.16	863,000	17.44	"	22.44	-157,000	54,000
6.	III_R.	-513,990	2.23	29.60	20.48	50.08	875,000	17.84	"	22.84	- 69,400	53,300
7.	III_L.	+425,810	2.28	31.73	21.52	53.25	883,000	18.24	"	23.24	+ 68,000	52,200
8.	II	-293,760	2.335	34.04	22.56	56.60	952,000	18.68	"	23.68	+233,000	51,100
9.	III_L.	-349,890	2.40	37.01	23.64	60.65	942,000	19.20	"	24.20	+436,000	49,500
10.	III_R.	+834,410	2.695	52.19	22.90	82.09	932,000	21.56	"	26.56	+ 711,000	47,500

The depth of arch rib at point 10 is increased 6 inches, by increasing the radius of the extradoes; this being done to bring the maximum combined stresses at this point within the allowable limits.

Note:- Subscripts after Case III refer to the right or left half of the arch rib.

QUANTITIES FOR COMPUTING STEEL FIBER STRESS.

Point	Case	M	u_1	I	N	A	M_{Temp}	N_{Temp}
1	II	+240,440	1.77	37.59	892,000	21.16	-344,000	55,000
2	III R.	-375,650	1.805	39.73	845,000	21.44	-333,000	54,700
3	III R.	-433,550	1.84	42.11	852,000	21.72	-300,000	54,500
4.	III R.	-510,690	1.88	44.42	852,000	22.04	-245,000	54,300
5	III L.	+619,800	1.93	47.16	863,000	22.44	-157,000	54,000
6	III R	-513,990	1.98	50.08	875,000	22.84	-69,400	53,300
7	III L.	+425,810	2.03	53.25	883,000	23.24	+68,000	52,200
8	II.	-293,760	2.085	56.60	952,000	23.68	+233,000	51,100
9	III L.	-349,890	2.15	60.65	942,000	24.20	+436,000	49,500
10.	III R.	+834,410	2.445	82.09	932,000	26.56	+711,000	47,500

CONCRETE.

MAXIMUM & MINIMUM FIBER STRESSES.

Point	Maximum Fiber Stress	Location of Fiber.	Minimum Fiber Stress	Location of Fiber	Temperature Stress		Resultant Stress.	
					Either Fiber	Either Fiber	Maximum	Minimum
1	-383	Upper	-203	Lower	-145	+110	-528	-93
2	-408	Lower	-139	Upper	-137	+99	-545	-40
3	-420	"	-122	"	-121	+86	-541	-36
4	-439	"	-98	"	-99	+65	-538	-33
5	-465	Upper	-68	Lower	-67	+34	-532	-34
6	-425	Lower	-107	Upper	-38	0	-463	-107
7	-390	Upper.	-137	Lower	0	-36	-426	-137
8	-362	Lower	-194	Upper	+52	-82	-444	-142
9	-368	"	-174	"	+105	-134	-502	-69
10	-433	Upper.	-54	Lower	+149	-174	-607	+95

− Compression.
+ Tension.

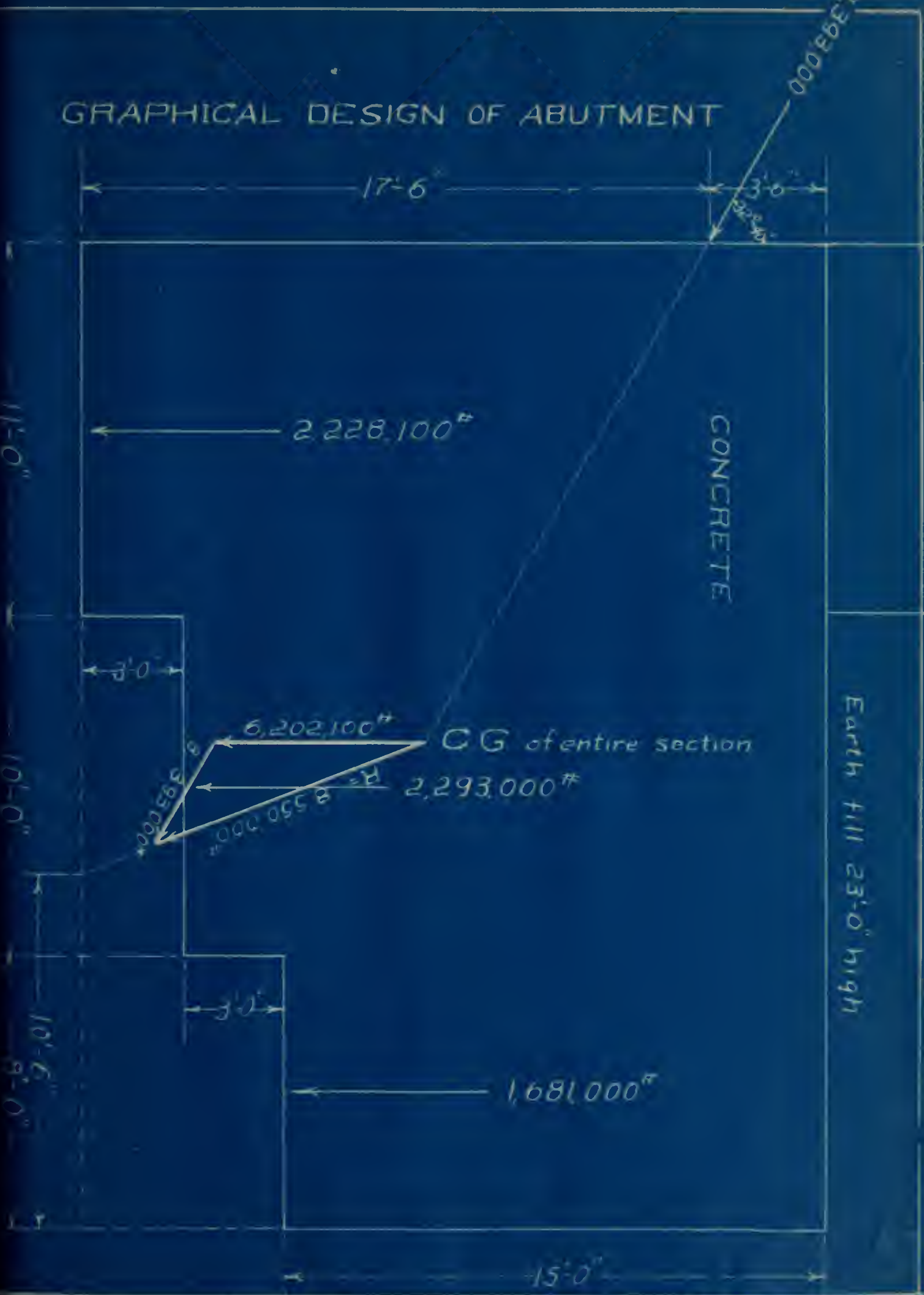
STEEL

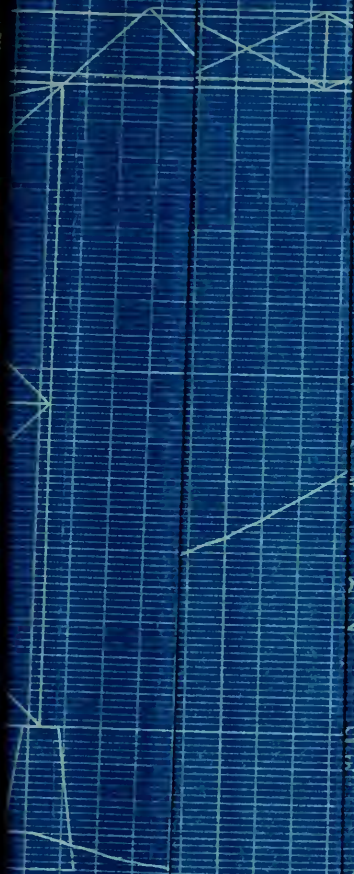
MAXIMUM & MINIMUM FIBER STRESSES

Point	Maximum Fiber Stress	Location of Fiber	Minimum Fiber Stress	Location of Fiber	Temperature Stress Either Fiber	Temperature Stress Either Fiber	Resultant Stress Maximum	Resultant Stress Minimum
1	-5580	Upper	-3210	Lower	-1,959	+1,417	-7539	-1793
2	-5875	Lower	-2325	Upper	-1,840	+1,310	-7715	-1015
3	-6050	"	-2,100	"	-1,624	+1,103	-7674	-997
4	-6,270	"	-1 770	"	-1,340	+826	-7610	-944
5	-6 650	Upper	-1360	Lower	-920	+420	-7570	-940
6	-6 100	Lower	-1870	Upper	-528	+43	-6628	-1827
7	-5 650	Upper	-2,270	Lower	-503	+36	-6153	-2234
8	-5290	Lower	-3040	Upper	-1,118	+670	-6408	-2370
9	-5350	"	-2 770	"	-1827	+1400	-7177	-1370
10	-6240	Upper	-1 070	Lower	-2395	+2022	-8635	+952

- Compression.
+ Tension.

GRAPHICAL DESIGN OF ABUTMENT





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RING DEPARTMENT

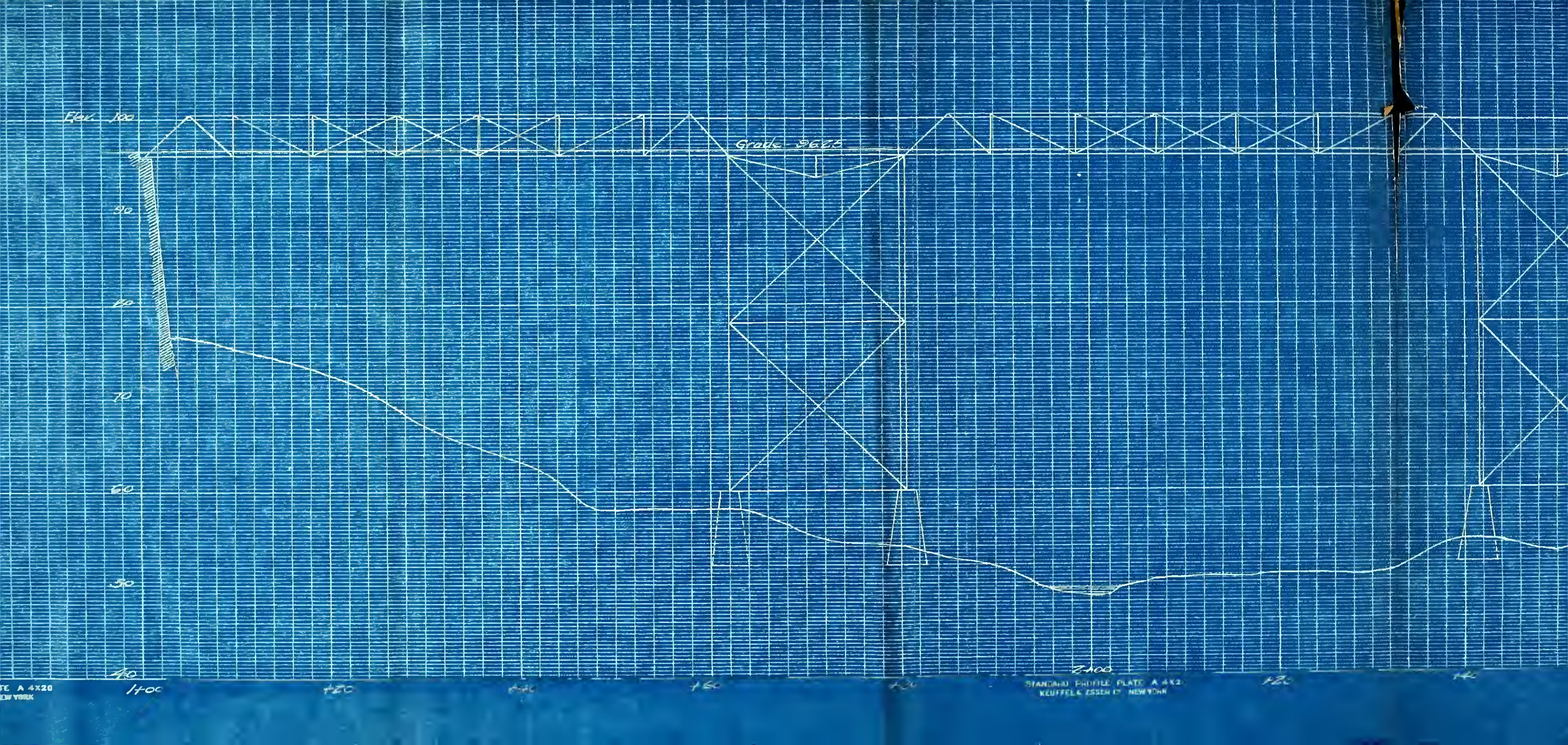
RTH SHERIDAN ROAD
AN, ILLINOIS

E FEB 4, 1911
& H.L.B

IS of {H. L. Butler
Schuyler M. Arnold

80

80



Elev. 100
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70
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50
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+20
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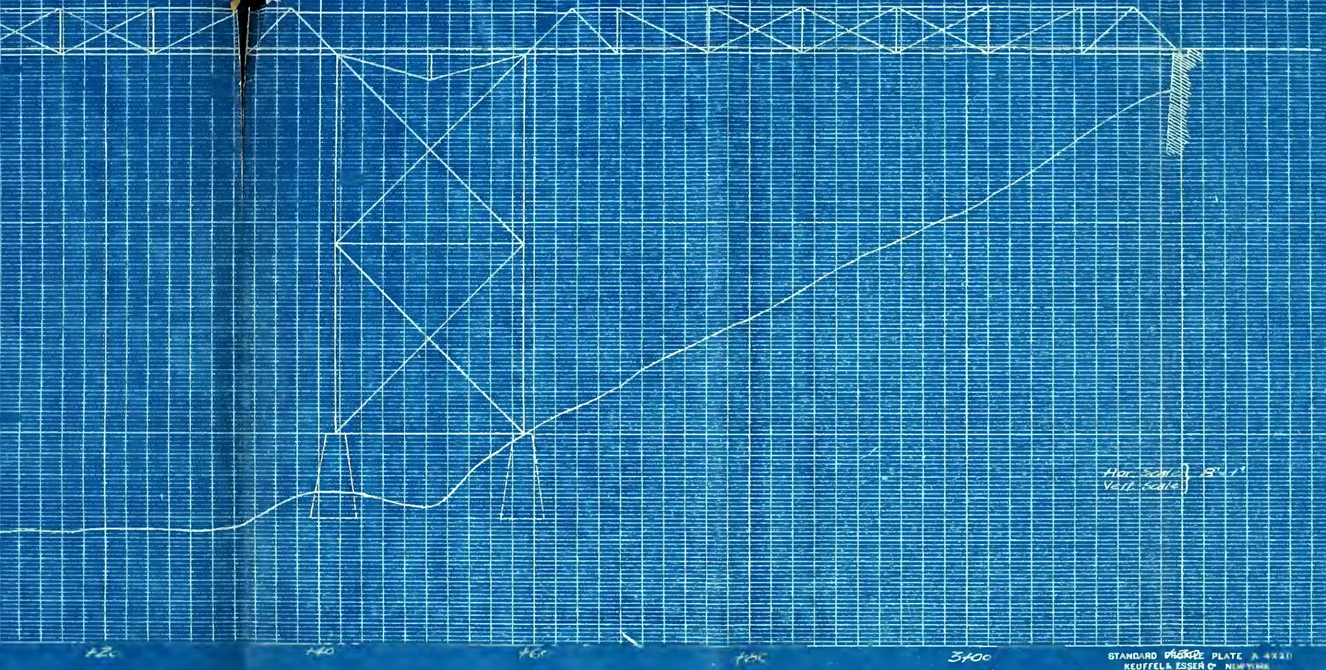
+40
STANDARD PROFILE PLATE A 4X2
KEUFFEL & ESSER CO NEW YORK

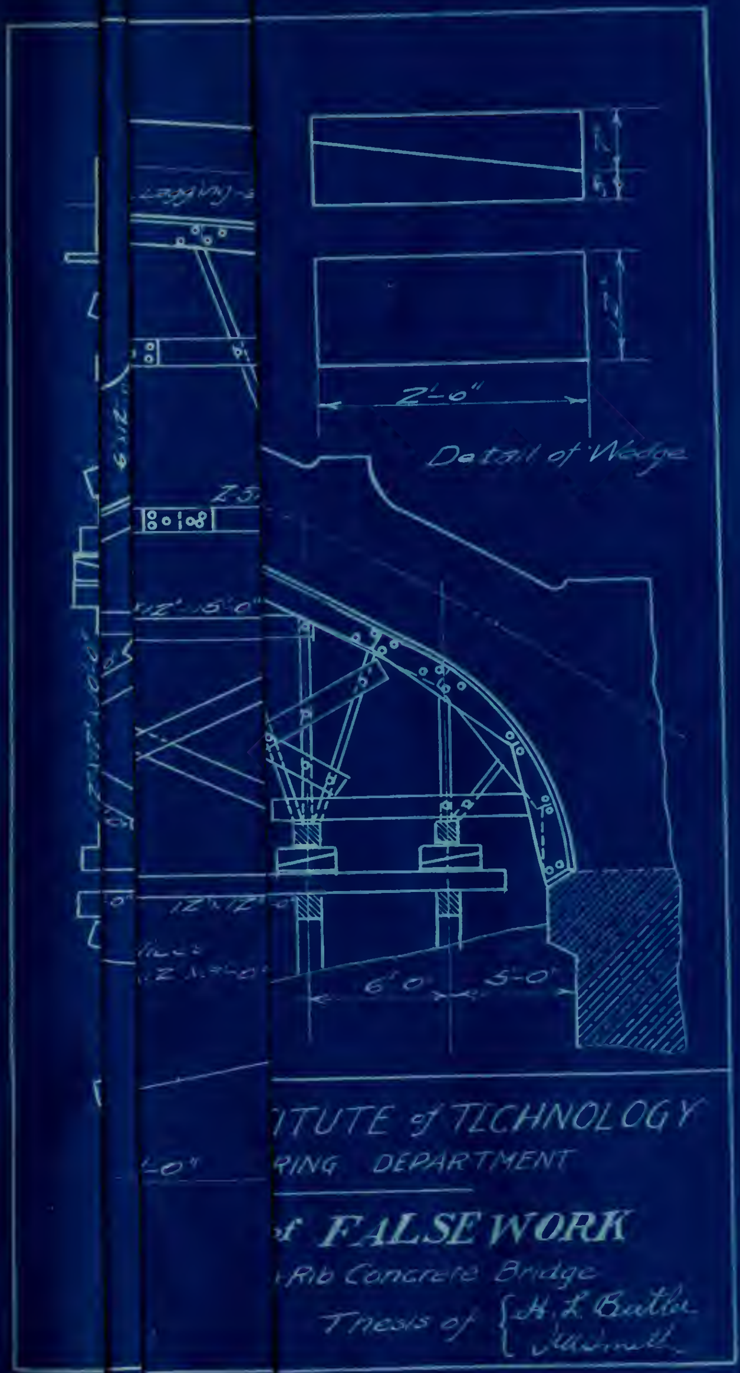


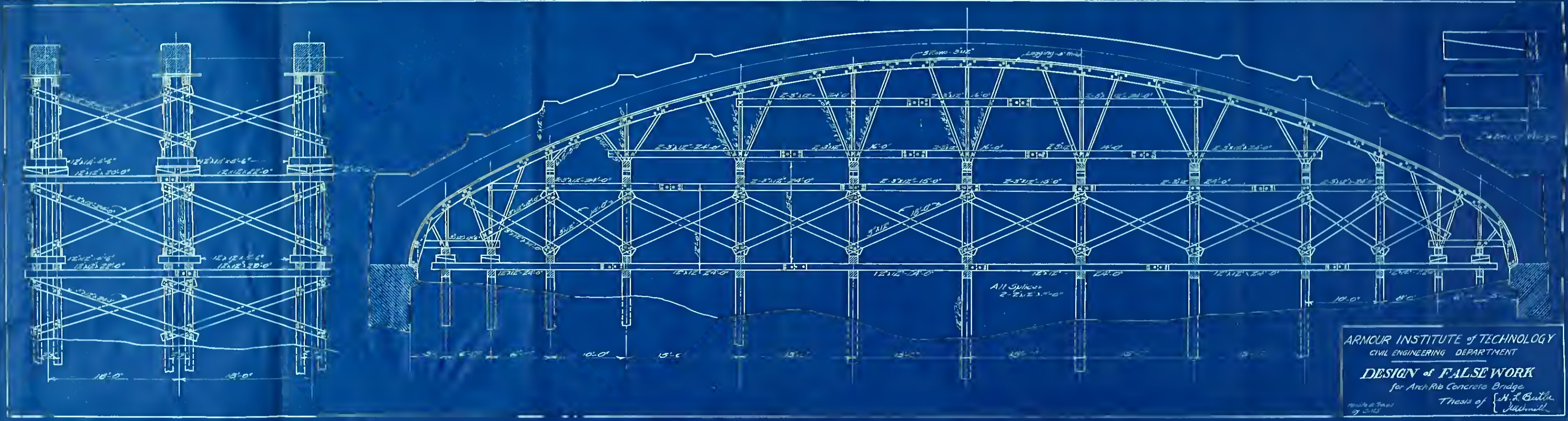
ARMOUR INSTITUTE of TECHNOLOGY
CIVIL ENGINEERING DEPARTMENT

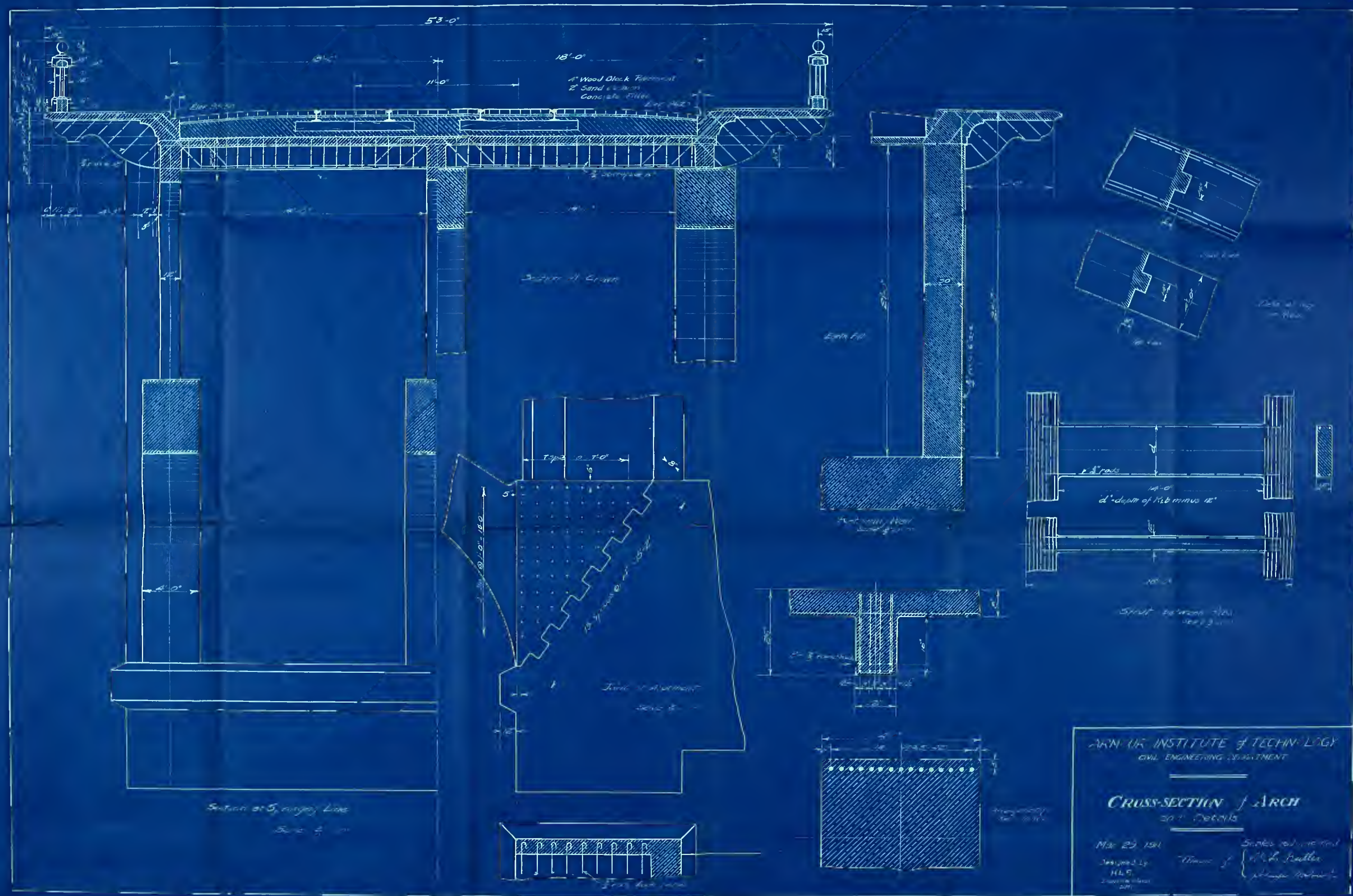
PROFILE OF NORTH SHERIDAN ROAD
WAUKEGAN, ILLINOIS

SURVEY MADE FEB. 4, 1911
BY S.M.S. & H.L.B.

Thesis of H. L. Butler, Schuyler M. Arnold

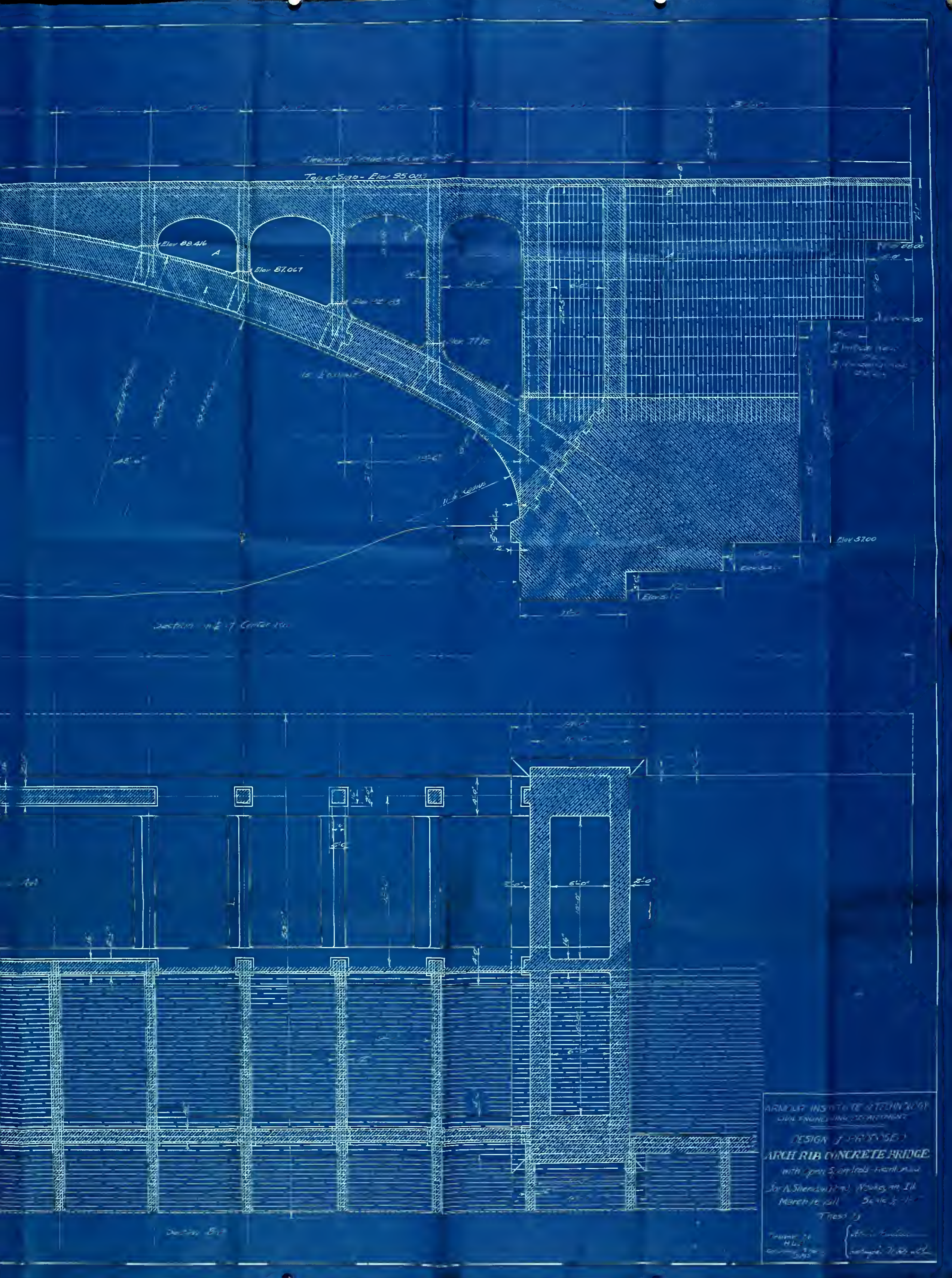

Lagging
2'-0"
Detail of Wedge
12"x12" 15'-0"
12"x12"
6'-0"
5'-0"
ITUTE of TECHNOLOGY
RING DEPARTMENT
FALSEWORK
Rib Concrete Bridge
Thesis of H. L. Butler

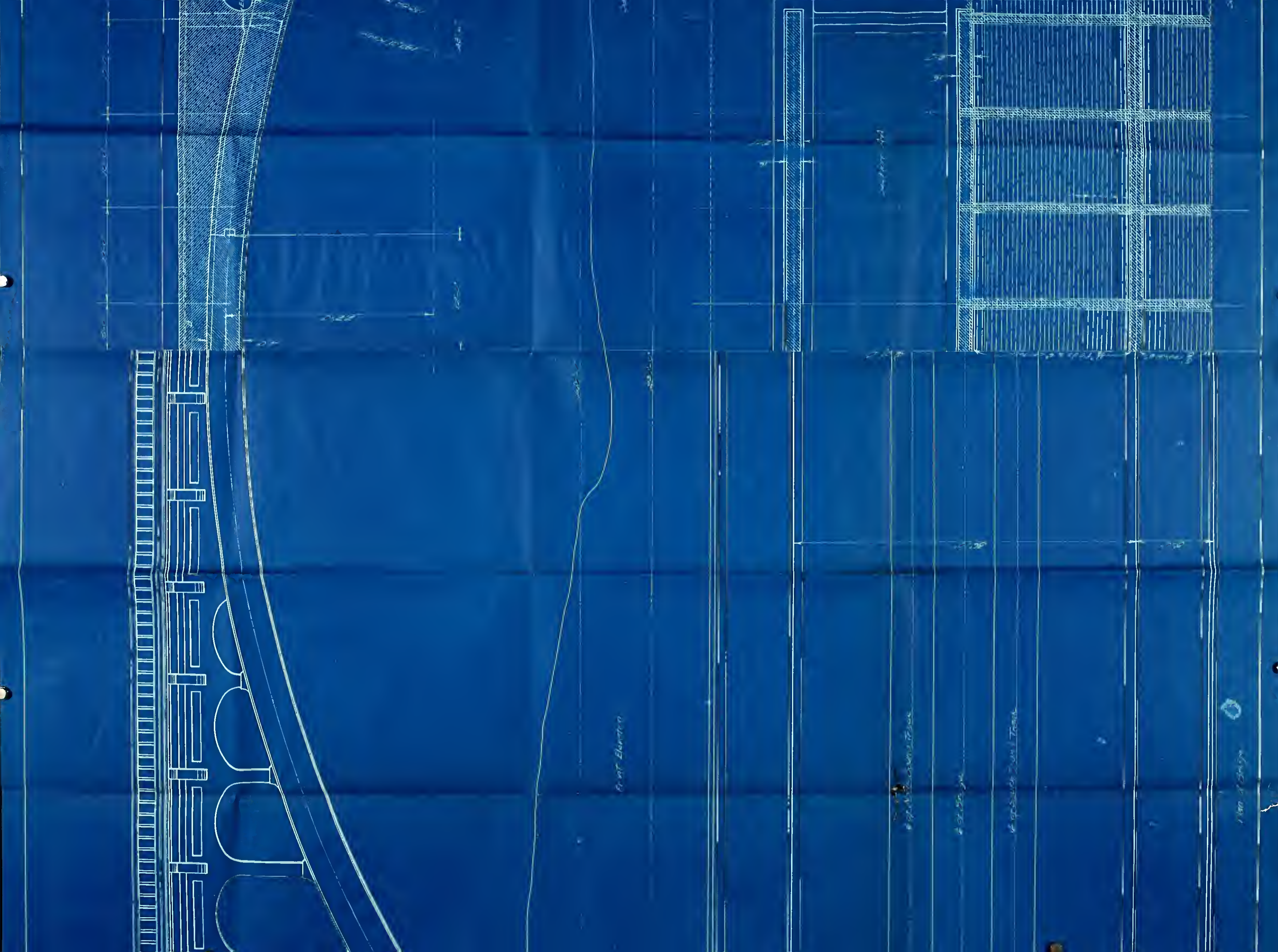

ARMOUR INSTITUTE of TECHNOLOGY
CIVIL ENGINEERING DEPARTMENT
DESIGN of FALSEWORK
for Arch Rib Concrete Bridge
Thesis of H. L. Butler
All Splices
18'-0"
10'-0"
15'-0"

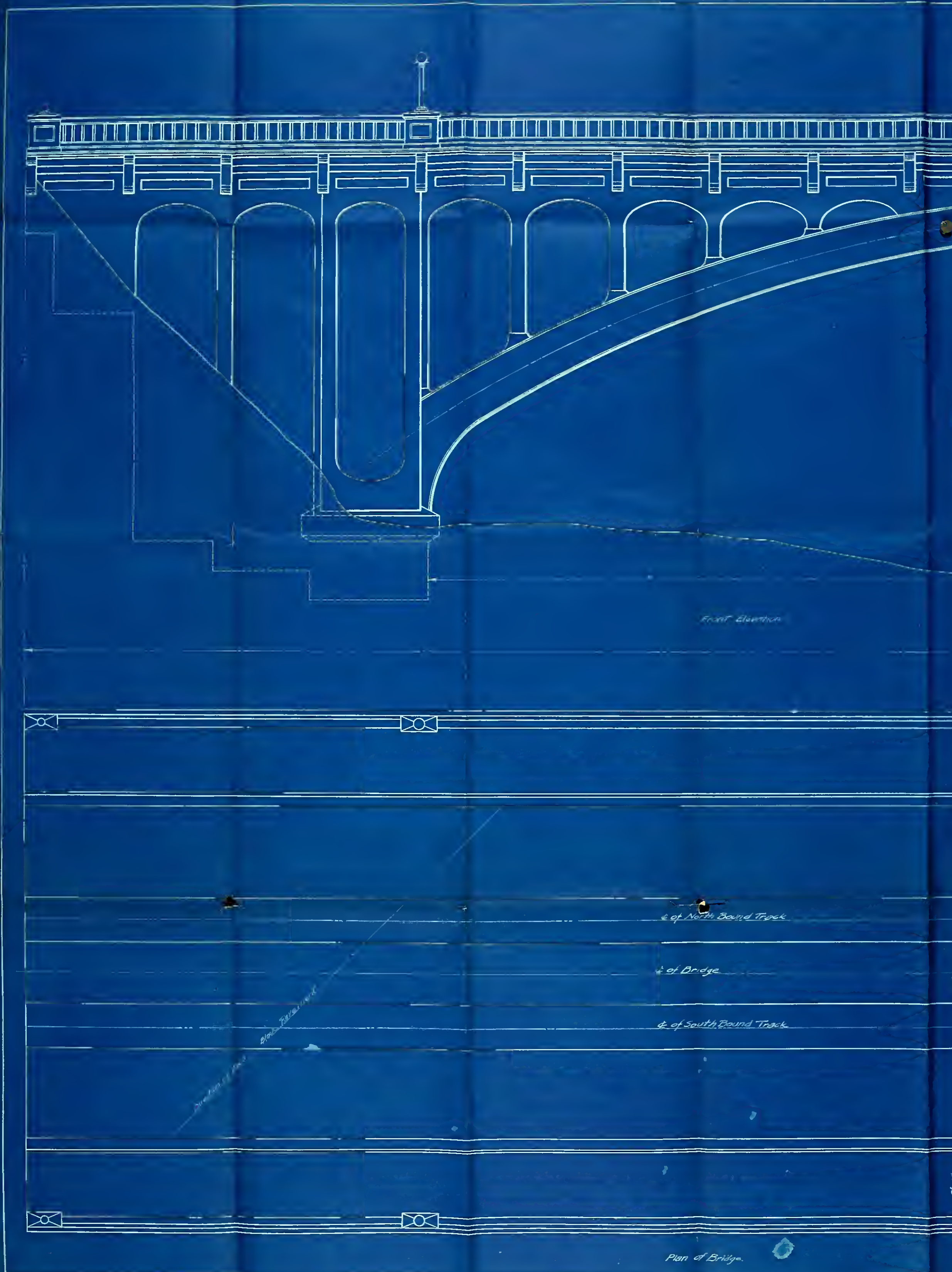


53'-0"
18'-0"
11'-0"
4" Wood Block Pavement
2" Sand Cushion
Concrete Filler
Section at Crown
Earth Fill
4'-0"
Joint at Abutment
14'-0"
"d" = depth of Rib minus 12"
Section at Springing Line
ARMOUR INSTITUTE of TECHNOLOGY
CIVIL ENGINEERING DEPARTMENT
CROSS-SECTION of ARCH
and Details
Designed by
H.L.S.


Elev 5700


Top of Slab - Elev 95.083
Elev 88.416
A
Elev 87.067
Elev 77.15
Elev 57.00
ARMOUR INSTITUTE OF TECHNOLOGY
CIVIL ENGINEERING DEPARTMENT
DESIGN of PROPOSED
ARCH RIB CONCRETE BRIDGE
with Open Spandrels
Thesis by


Front Elevation



Plan of Bridge.